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NATIONAL DAM SAFETY PROGRAM. STEPHENS LAKE DAM, (MO 11172), MIS-ETC(U)

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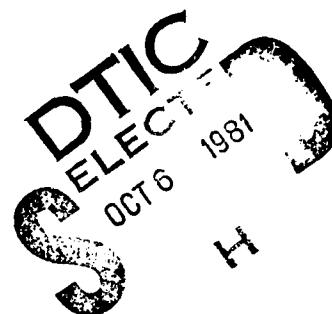
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STEPHENS LAKE DAM
BOONE COUNTY, MISSOURI
MO. 65172



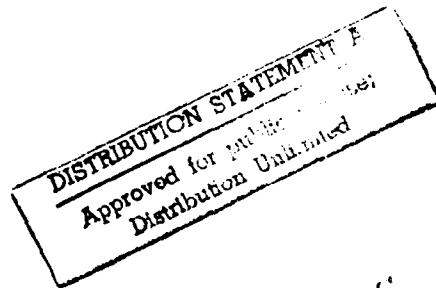
PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM



United States Army
Corps of Engineers

... Serving the Army
... Serving the Nation

St. Louis District



PREPARED BY: U. S. ARMY ENGINEER DISTRICT, ST. LOUIS

FOR: STATE OF MISSOURI

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DECEMBER 1980

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REPLY TO
ATTENTION OF

DEPARTMENT OF THE ARMY
ST. LOUIS DISTRICT, CORPS OF ENGINEERS
210 TUCKER BOULEVARD, NORTH
ST. LOUIS, MISSOURI 63101

SUBJECT: Stephens Lake Dam Phase I Inspection Report

This report presents the results of field inspection and evaluation of the Stephens Lake Dam (MO 11172).

It was prepared under the National Program of Inspection of Non-Federal Dams.

This dam has been classified as unsafe, non-emergency by the St. Louis District as a result of the application of the following criteria:

- a. Spillway will not pass 50 percent of the Probable Maximum Flood without overtopping the dam.
- b. Overtopping of the dam could result in failure of the dam.
- c. Dam failure significantly increases the hazard to loss of life downstream.

SUBMITTED BY:

SIGNED

Chief, Engineering Division

24 FEB 1981

Date

APPROVED BY:

SIGNED

Colonel, CE, District Engineer

25 FEB 1981

Date

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STEPHENS LAKE DAM
BOONE COUNTY, MISSOURI

MISSOURI INVENTORY NO. 11172

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

PREPARED BY
CONSOER, TOWNSEND AND ASSOCIATES, LTD.
ST. LOUIS, MISSOURI
AND
PRC ENGINEERING CONSULTANTS, INC.
ENGLEWOOD, COLORADO
A JOINT VENTURE

UNDER DIRECTION OF
ST. LOUIS DISTRICT, CORPS OF ENGINEERS
FOR
GOVERNOR OF MISSOURI

DECEMBER 1980

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam: Stephens Lake Dam, Missouri Inv. No. 11172
State Located: Missouri
County Located: Boone
Stream: An unnamed tributary of Hinkson Creek
Date of Inspection: July 11, 1980

Assessment of General Condition

Stephens Lake Dam was inspected by the engineering firms of Consoer, Townsend and Associates, Ltd. and PRC Engineering Consultants, Inc. (A Joint Venture) of St. Louis, Missouri according to the U. S. Army Corps of Engineers' "Recommended Guidelines for Safety Inspection of Dams" and additional guidelines furnished by the St. Louis District of the Corps of Engineers. Based upon the criteria in the guidelines, the dam is in the high hazard potential classification, which means that loss of life and appreciable property loss could occur in the event of failure of the dam. Within the estimated damage zone of four miles downstream of the dam, there are thirteen dwellings, one shopping center, apartment houses, and a trailer court which may be subjected to flooding, with possible damage and/or destruction, and possible loss of life. Stephens Lake Dam is in the small size classification since it is 23 feet high, and impounds more than 50 acre-feet but less than 1,000 acre-feet of water.

The inspection and evaluation of the consultant's inspection team indicate that the spillway of Stephens Lake Dam does not meet the criteria set forth in the guidelines for a dam having the above size and hazard potential. Stephens Lake Dam being a small size dam with a high hazard potential is required by the guidelines to pass from one-half of the Probable Maximum Flood to the Probable Maximum Flood without overtopping. Considering the large number of inhabited dwellings located downstream of the dam, the PMF is considered the appropriate spillway design flood for Stephens Lake Dam. The Probable Maximum Flood is defined as the flood discharge that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in the region. It was determined that the reservoir/spillway system can accommodate approximately ~~50~~⁴⁵ percent of the Probable Maximum Flood without overtopping the dam. Our evaluation also indicates that the reservoir/spillway system can accommodate the one-percent chance flood (100-year flood) without overtopping.

Stephens Lake Dam and its appurtenant structures appear to be in fair condition due to what appears to be the possibility of past piping of the embankment material along the spillway pipe. This condition is considered to be a significant deficiency and has already caused some damage to the spillway and the dam embankment.

Other deficiencies noted by the inspection team were: the erosion due to wave action on the upstream slope, rodent holes in the embankment, the small shrubs growing in the wave eroded area, a need for periodic inspection by a qualified engineer and a lack of maintenance schedule. The lack of seepage and stability analyses on record is also a deficiency that should be corrected.

It is recommended that the owner take action to correct or control the deficiencies described above.



Walter G. Shifrin, P.E.

Overview of Stephens Lake Dam



NATIONAL DAM SAFETY PROGRAM

STEPHENS LAKE DAM, I.D. No. 11172

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PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

STEPHENS LAKE DAM, Missouri Inv. No. 11172

SECTION 1: PROJECT INFORMATION

1.1 General

a. Authority

The Dam Inspection Act, Public Law 92-367 of August, 1972, authorizes the Secretary of the Army, through the Corps of Engineers, to initiate a national program of dam inspections. Inspection for Stephens Lake Dam was carried out under Contract DACW 43-80-C-0094 between the Department of the Army, St. Louis District, Corps of Engineers, and the engineering firms of Consoer, Townsend & Associates, Ltd., and PRC Engineering Consultants, Inc. (A Joint Venture), of St. Louis, Missouri.

b. Purpose of Inspection

The visual inspection of Stephens Lake Dam was made on July 11, 1980. The purpose of the inspection was to make a general assessment as to the structural integrity and operational adequacy of the dam embankment and its appurtenant structures.

c. Scope of Report

This report summarizes available pertinent data relating to the project, presents a summary of visual observations made during the field inspection, presents an assessment of hydrologic and hydraulic conditions at the site, and the structural adequacy of

the various project features and assesses the general condition of the dam with respect to safety.

Subsurface investigations, laboratory testing and detailed analyses were not within the scope of this study. No warranty as to the absolute safety of the project features is implied by the conclusions presented in this report.

It should be noted that in this report reference to left or right abutments is viewed as looking downstream. Where left abutment or left side of the dam is used in this report, this also refers to the north abutment or side, and right abutment or right side to the southwest abutment or side.

d. Evaluation Criteria

The inspection and evaluation of the dam is performed in accordance with the U.S. Army Corps of Engineers' "Recommended Guidelines for Safety Inspection of Dams" and additional guidelines furnished by the St. Louis District office of the Corps of Engineers for Phase I Dam Inspection.

1.2 Description of the Project

a. Description of Dam and Appurtenances

The following description is based upon observations and measurements made during the visual inspection and from conversations with Mr. Marion Henley, Director of Buildings and Grounds for Stephens College. No design drawings were available for this dam or appurtenant structures.

The dam is a compacted earthfill structure between earth abutments. The measured top width is 14 feet and the length along the axis is 648 feet. A plan and elevation of the dam are shown on Plate 2 and Photos 1 through 3 show views of the dam. The alignment

of the dam is generally straight along the middle 400 feet of the embankment with an average curvature in the upstream direction of 15 degrees in the last 100 feet on each extreme of the dam. The top of dam has a minimum elevation of 691.3 feet above mean sea level (M.S.L.) which occurs at about the left 1/3 point and the maximum structural height of the embankment which occurs at approximately the right 1/3 point was measured to be 23 feet. At the location of the minimum top of dam elevation, the top of dam slopes upward to each abutment with a rise in elevation of 1 foot. The top of dam is used as an access road for light maintenance equipment.

The downstream slope of the embankment was measured to be 1V on 2.25H. It was not possible to accurately measure the upstream slope because of wave erosion on the face and a near horizontal, riprapped bench at the water surface. However, the measurements made over the short unaltered upstream slope indicated the upstream slope to be 1V to 1.5H. Except for the riprapped bench and wave eroded face of the upstream slope, the entire exposed embankment is protected by a dense short grass cover.

There is only one spillway at the damsite which consists of a concrete side channel connected to a vitrified clay pipe which passes through the embankment. The side channel structure consists of a rectangular shaped concrete box which is 10.6-feet long, 2.5-feet wide and 2.5-feet deep (see Photo 5). The control section of the channel is located on the south side of the box and has an assumed crest elevation of 689.0 feet above M.S.L., which places the crest 6 inches below the top of the rest of the structure. The clay pipe is 24 inches in diameter and about 37 feet long. The pipe is laid through the embankment on a 10 percent grade. A 6-inch high, wood framed structure with a wire screen was provided at the entrance to the side channel as a fish screen. The spillway is located approximatley 38 feet to the right of the left abutment/embankment contact.

A 4-inch diameter siphon pipe was provided at the damsite to drain the reservoir if needed. The siphon consists of a 4-inch steel pipe which is controlled by a 4-inch gate valve located on the upstream side of the system (see Photo 9). According to Mr. Henley, the siphon was last used in 1955 to lower the reservoir. The siphon is located about 150 feet to the right of the left abutment.

An electric powered, vertical submersible centrifugal turbine pump was installed at the damsite (see Photo 10). The purpose of the pump is to pump groundwater into the reservoir to help keep the reservoir at a desired level. The pumphouse is located on the right side of the reservoir.

b. Location

Stephens Lake Dam is located in Boone County in the State of Missouri, and crosses an unnamed tributary of Hinkson Creek. The dam is located on the east edge of the City of Columbia. The Stephens Lake Dam location on the 7.5 minute series of the U.S. Geological Survey maps is found in Section 7 of Township 48 North, Range 12 West, of the Columbia, Missouri Quadrangle Sheet.

c. Size Classification

The impoundment of Stephens Lake Dam is less than 1,000 acre-feet but more than 50-acre feet, and its height is 23 feet. Therefore, the size is determined to fall in the "small" category, according to the "Recommended Guidelines for Safety Inspection of Dams" by the U.S. Department of the Army, Office of the Chief Engineer.

d. Hazard Classification

The dam has been classified as having a "high" hazard potential in the National Inventory of Dams, on the basis that in the event of failure of the dam or its appurtenances, excessive damage could occur to downstream property, together with the possibility of the loss of life. The findings of the consultant's inspection team concur with this classification. There are thirteen dwellings, apartment houses, a shopping center and a trailer court within the estimated damage zone, extending four miles downstream of the dam.

e. Ownership

Stephens Lake Dam is owned by Stephens College of Columbia, Missouri. All correspondence is directed to Mr. Marion Henley, Director of Buildings and Grounds, Stephens College. The mailing address is as follows: 1200 East Broadway, Columbia, Missouri, 65215.

f. Purpose of Dam

At present the Stephens Lake is used only for recreation. However, originally the lake was built for stock watering purposes. At that time, according to Mr. Henley, the impoundment was much smaller than the present impoundment.

g. Design and Construction History

The information on the design and construction of the dam, as described below, was given to the inspection team by Mr. Henley. The original dam was built around the turn of the century for stock watering purposes. The original dam and lake were much smaller than the present dam and lake. Stephens College purchased the property in the late 1920's and the lake was enlarged in 1939 by increasing the size of the dam. Since the watershed area was not

sufficient to support the enlarged lake, a deep well was also drilled at the same time and a pump installed to pump water into the reservoir to help keep the reservoir at a desired level. The lake and dam were probably constructed without any engineering design and supervision. The lake level was lowered in 1955 and a larger swimming area was blasted out of the bedrock on the south rim of the reservoir. The spillway was also constructed at this time.

h. Normal Operational Procedures

Normal procedures is to allow the reservoir to remain as full as possible with the water level being controlled by rainfall, runoff, evaporation, the elevation of the spillway crest, and periodic supply of groundwater from the well near the lake.

1.3 Pertinent Data

a. Drainage Area (acres): 38

b. Discharge at Damsite

Estimated experienced maximum flood (cfs): 4

Estimated ungated spillway capacity with
reservoir at top of dam elevation (cfs): 42

c. Elevation (Feet above MSL)

Top of dam (minimum):. 691.3

Spillway crest*: 689

Normal Pool: 689

Maximum Experienced Pool:. 689.25

Observed Pool: 688.3

d. Reservoir

Length of pool with water surface
at top of dam elevation (feet):.. 1100

e. Storage (Acre-Feet)

Top of dam (minimum):. 89

Spillway crest: 63

Normal Pool: 63

Maximum Experienced Pool:. 65.5

Observed Pool: 55

f. Reservoir Surfaces (Acres)

Top of dam (minimum):. 12

Spillway crest: 10

Normal Pool: 10

Maximum Experienced Pool:. 10.3

Observed Pool: 9.8

g. Dam

Type: Rolled, earthfill
Length: 648 feet
Structural Height: 23 feet
Hydraulic Height**: 23 feet
Top width: 14 feet
Side slopes:
 Downstream 1V on 2.25H
 Upstream 1V on 1.5H (Above the water surface)
Zoning: Unknown
Impervious core: Unknown
Cutoff: Unknown
Grout curtain: Unknown
Freeboard above
normal reservoir level: 2.3 feet (Minimum)
Volume: 23,700 cu. yds. (Estimated)

h. Diversion and Regulating Tunnel None

i. Spillway

Type: Side channel and culvert
combination
Length of crest: 10.6 feet
Crest Elevation (feet above MSL): . . . 689

j. Regulating Outlets

Type: 4-inch siphon (Inoperable)
Location: 150 feet to the right of the
left abutment
Length: Unknown
Closure: 4-inch gate valve
Maximum Capacity: Unknown

* The elevation of the spillway crest is assumed from the U.S.G.S.
Columbia, Missouri Quadrangle topographic map. The elevation of
other features of the dam are obtained by using this elevations and
field measurements.

** The hydraulic height of the dam is the vertical distance from the lowest point on the downstream toe to the top of dam or the maximum water surface, if below the top of dam.

SECTION 2: ENGINEERING DATA

2.1 Design

No design data are available for the dam and appurtenant structures. Mr. Henley of Stephens College did provide a one sheet survey plan showing contour lines and elevations which was drawn about 1930 by W. B. Cauthorn, a local engineer. He also made available a "Pump Installation Report" dated May 10, 1963 which lists the characteristics of the pump and well.

2.2 Construction

No construction records or data are available for Stephens Lake Dam.

2.3 Operation

No operational records are available for Stephens Lake Dam.

2.4 Evaluation

a. Availability

No design drawings, design computations, construction data, or operation data are available. Also, no pertinent data were available for review of hydrology, spillway capacity, flood routing through the reservoir, outlet capacity, slope stability, or foundation conditions. Seepage and stability analyses comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available, which is considered a deficiency.

b. Adequacy

The lack of engineering data did not allow a definitive review and evaluation. Therefore, the adequacy of this dam could not be assessed from the standpoint of reviewing and evaluating design, operation and construction data, but is based primarily on visual inspection, past performance history, and sound engineering judgment.

Seepage and stability analyses comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available, which is considered a deficiency. These seepage and stability analyses should be performed for appropriate loading conditions (including earthquake loads) and made a matter of record.

c. Validity

No valid engineering data relating to the design and construction of the dam are available for Stephens Lake Dam.

SECTION 3: VISUAL INSPECTION

3.1 Findings

a. General

A visual inspection of the Stephens Lake Dam was made on July 11, 1980. The following persons were present during the inspection:

Name	Affiliation	Disciplines
Dr. M.A. Samad	PRC Engineering Consultants, Inc.	Project Engineer, Hydraulics and Hydrology
Mark Haynes, P.E.	PRC Engineering Consultants, Inc.	Civil and Mechanical
Razi Quraishi, R.P.G.	PRC Engineering Consultants, Inc.	Geology
Zoran Batchko	PRC Engineering Consultants, Inc.	Soils
Kevin J. Blume	Consoer, Townsend & Assoc., Ltd.	Civil and Structural
Joe Kellett	Corps of Engineers	
Randall Dreiling	Corps of Engineers	

David Busse Corps of Engineers

Wayne Richter Corps of Engineers

Mr. Marion Henley Stephens College

Specific observations are discussed below.

b. Dam

The top and the downstream slope of the dam have a well maintained grass cover which adequately protects the embankment material against surface erosion. Rodent holes less than 1-1/2 inches in diameter were observed on the downstream slope (see Photo 4). According to Mr. Henley, the dam has never been overtopped and no evidence indicating the contrary was observed.

The upstream slope has riprap protection extending from about 2.5 feet below the top of dam to below the water surface where sloughing of the riprap is prevented by batter boards (see Photo 1). The upstream slope has been eroded by wave action. The slope of the exposed riprapped portion of the upstream slope was measured to be 5° (nearly horizontal) while the scarp due to wave action is typically 45° (1V on 1H) or steeper. The exposed eroded face is generally on the order of 18 inches high and vegetated by small shrubs. The exposed embankment material is a dark gray, moderately plastic silty clay.

There is no evidence of seepage or leakage through or below the dam, except for two large voids in the downstream face immediately adjacent to the spillway pipe (see Photo 7). These voids appear to be due to leakage along the spillway pipe, as further described in Section 3.1.d.

No signs of past or present instability were seen on the embankment except for the wave eroded upstream slope near the crest.

Both abutments slope gently upward from the top of dam. No instabilities, seepage, or erosion were observed on either abutment.

c. Project Geology and Soils

(1) Project Geology

The damsite is located on an unnamed tributary of Hinkson Creek in the Dissected Till Plains Section of the central Lowland Physiographic Province. Loess-mantled Kansas Drift covers the surface of most of the Dissected Till Plains Section. This section is distinguished from the Young Drift Section to the north and from the Till Plains on the east by the stage it has reached in the post-glacial erosion cycle. Broadly generalized, this section is a nearly flat till plain submature to mature in its erosion cycle.

The topography at the damsite is rolling to hilly with gentle slopes. Elevations of the ground surface range from 690 feet above M.S.L. at the damsite to 750 feet above M.S.L. approximately 0.5 mile from the damsite. The reservoir slopes at the southern side of the reservoir are in the range of 15° to 26° from the horizontal, between 10° to 20° from the horizontal at the northern side, and in the range of 7° from horizontal at the western side. The reservoir slopes appear to be stable and free of any potential slide activity. The area near the damsite is covered with slope wash of glacial-fluvial deposits and loess.

The regional bedrock geology beneath the glacial outwash deposits in the damsite area as shown on the Geologic Map of Missouri (1979), (see Plate 3), consists of Pennsylvanian undifferentiated rocks, Pennsylvanian Marmaton-Cherokee Group (cyclic deposits of shale, limestone, and sandstone), Mississippian age Burlington

Limestone (cherty, grayish brown sandy limestone), Devonian age rocks of the Sulphur Springs Group (Glen Park Limestone and Grassy Creek Shale), and the Ordovician age rocks consisting of St. Peter Sandstone and Powell Dolomite. The predominant bedrock near the site underlying the glacial-fluvial deposits are the Marmaton-Cherokee Group rocks and the Burlington Limestone.

Outcroppings of Pennsylvanian Marmaton Group rocks consisting of slightly weathered to unweathered, whitish gray, fine to medium grained, hard limestone are exposed in a hill adjacent to the northeast rim of Stephens reservoir and at the swimming area (see Photo 11). These rocks are horizontally bedded with a rectangular jointing pattern. Inlet and outlet areas to the reservoir of the unnamed tributary of Hinkson Creek contain Quaternary alluvium.

No faults have been identified in the vicinity of the damsite. The closest trace of a fault to the damsite is the Fox Hollow fault nearly 15 miles southwest of the damsite. The Fox Hollow fault had its last movement in post-Mississippian time. Thus, the fault has no effect on the dam.

Stephens Lake, Dam consists of an earthfill embankment (dark gray to brown silty clay), with a side channel/clay pipe combination spillway located near the left abutment. Based on the available data, conversations with Mr. Marion Henley and the visual inspection, the embankment rests on the Pennsylvanian Marmaton Group rock consisting of unweathered whitish gray, fine to medium grained, hard limestone. The entire spillway system rests on the compacted embankment fill.

(2) Project Soils

According to the "Missouri General Soil Map and Soil Association Description" published by the Soil Conservation Service, the materials in the general area of the dam belong to the soil series of Sharpsburg-Pole-Sogn-Snead in the Deep Loess and Drift

family. The soils were basically formed from loess and the weathering of calcareous clay shale and limestone. The permeability of these soils ranges from moderate to slow.

Materials were removed from below the vegetative cover on the downstream and upstream embankment slopes. The material removed from the embankment near the left abutment and representative of the left most 20 feet of the embankment appeared to be a yellowish brown, low plasticity sandy clay. Based upon the Unified Soil Classification System, the soil would probably be classified as a CL. This soil type generally has the following characteristics: semipervious with a coefficient of permeability less than 500 feet per year, medium to high shear strength, and a low to intermediate resistance to piping. The materials removed from and representative of the remainder of the embankment appeared to be a dark gray silty clay with a trace of fine to coarse sand. Based upon the Unified Soil classification System, the soil would probably be classified as a CL. This soil type generally has the following characteristics: impervious with a coefficient of permeability less than 100 feet per year, medium to high shear strength and an intermediate resistance to piping.

d. Appurtenant Structures

(1) Spillway

The side channel structure appeared to be stable with no major problems apparent. However, some minor leaching and cracking of the concrete was observed. The stability of the clay pipe appeared to be in jeopardy. It appears that water from the upstream end has been flowing along the outside of the pipe and has carried embankment material along with it. Two large holes to the right of the outlet of the pipe (see Photo 7), a small depression on the downstream slope over the pipe and several cracks on the upstream slope over the pipe were observed which indicates the possibility that past piping of the embankment material along the pipe has

occurred. Mr. Henley believes that concrete was dumped near the inlet of the spillway to alleviate this problem (see Photo 5). The joints of the pipe were also misaligned which indicates that voids have possibly been created under the pipe allowing differential settlement of the sections of pipe to occur. At the outlet end of the pipe, a concrete apron was constructed which extends out from the end of the pipe a distance of 2 feet. At the end of the concrete apron, flows through the pipe will drop into what appears to be the top portion of a buried 5-foot diameter steel drum (see Photo 6). The steel drum appears to act as a stilling basin. Beyond the steel drum, the discharge channel for the spillway is riprapped for the short distance it travels before intersecting the downstream channel just downstream of the dam. The outlet end of the pipe does not appear to be undermined.

(2) Siphon

The siphon was inoperable on the day of inspection due to the fact that the downstream portion of the siphon was cut off at the top of dam making it impossible for the siphon to operate. It was also noted that hand wheel operator for the gate valve was missing. According to Mr. Henley, at one time the siphon pipe did extend down the downstream slope to the toe and that the downstream portion of the pipe was removed by maintenance personnel to help facilitate the mowing of the downstream slope. Mr. Henley stated that he has access to a portable pump which can be used to level the reservoir instead of using the siphon.

e. Reservoir Area

The reservoir water surface elevation at the time of inspection was 688.3 feet above M.S.L.

The surface area of the reservoir at normal water level is about 10 acres. The rim seems to be stable as no severely eroded areas were observed. The land around the reservoir slopes gently to the rim and is grass and/or tree covered. There are no homes built in close proximity to the reservoir.

f. Downstream Channel

The downstream channel, which carries flows from the spillway, is a narrow gulley which crosses a golf course immediately below the dam. The channel is approximately 3 feet wide, 2 feet deep and has nearly vertical side slopes. Some erosion was observed on the sides of the channel. Outside of the small channel the floodplain widens out considerably (see Photo 8).

3.2 Evaluation

The visual inspection revealed the following condition that was felt to pose a threat to the safety of the dam and the spillway and would warrant prompt attention.

It appears that piping of embankment material has occurred in the past along the spillway pipe. This is indicated by the two large voids near the outlet of the spillway, a small depression on the downstream slope of the spillway pipe, several cracks on the upstream slope over the spillway pipe, and the misalignment of the joints of the spillway pipe. The stability of the spillway pipe appears to be in jeopardy due to this condition and if the condition is allowed to progress, it can only be detrimental to the stability of the dam and the spillway.

The following items were observed that are not sufficiently significant to indicate a need for immediate remedial action; however, they could adversely affect the dam in the future.

1. The wave erosion on the upstream slope does not appear to affect the stability of the dam in its present condition. However, continual erosion of the slope can only be detrimental to the stability of the dam.
2. The small shrubs on the upstream face growing in the wave eroded area should be properly maintained. Large vegetation could hinder a comprehensive inspection of the dam and allow potential problems to go undetected.
3. The rodent holes observed on the embankment could jeopardize the safety of the dam. The holes created by the animals make avenues for possible piping.

SECTION 4: OPERATIONAL PROCEDURES

4.1 Procedures

There are no specific operational procedures which are followed at Stephens Lake Dam. When dry periods occur, water is pumped from the nearby well to keep the lake at a desired level.

4.2 Maintenance of Dam

The dam is maintained by workmen from the Buildings and Grounds Dept. of Stephens College. Mr. Marion Henley, Director of Buildings and Grounds, oversees the operation and maintenance of the lake and dam. At the time of inspection, the maintenance personnel were in the process of trying to trap the rodents which have burrowed into the embankment on the downstream slope. The top of dam and the embankment slopes are mowed regularly.

4.3 Maintenance of Operating Facilities

There are two operating facilities at the damsite. They are the vertical centrifugal turbine pump located on the south side of the reservoir and the siphon. The pump is maintained by personnel from the Buildings and Grounds Department of Stephens College. The siphon is inoperable.

4.4 Description of Any Warning System in Effect

The inspection team is not aware of any warning system in use at the damsite.

4.5 Evaluation

The maintenance for this dam is somewhat lacking. The corrective measures listed in Section 7 should be undertaken to improve the condition of the dam.

SECTION 5: HYDRAULIC/HYDROLOGIC

5.1

Evaluation of Features

a. Design Data

No hydrologic and hydraulic design data are available for Stephens Lake Dam. The sizes of physical features utilized to develop the stage-outflow relation for the spillway and overtopping of the dam were prepared from field notes and sketches prepared during the field inspection. The reservoir elevation-area data were based on the U.S.G.S. Columbia, Missouri Quadrangle topographic maps (7.5 minute series). The spillway and overtop release rates and the reservoir elevation-area data are presented in Appendix B.

The hydrologic soil group of the watershed was determined from information available in the U.S.D.A. Soil Conservation Service publication "Missouri General Soil Map and Soil Association Descriptions", 1979. The Probable Maximum Precipitation (PMP) used to determine the Probable Maximum Flood (PMF) was determined by using the U.S. Weather Bureau publication, "Hydrometeorological Report No. 33" (April 1956). The 100-year flood was derived from 100-year rainfall of Jefferson City, Missouri, supplied by the St. Louis District of the Corps of Engineers.

b. Experience Data

It is believed that records of reservoir stage or spillway discharge are not maintained for this site. However, according to Mr. Henley, the maximum reservoir level was approximately 3-inches above the crest of the spillway.

c. Visual Observations

Observations made of the spillway during the visual inspection are discussed in Section 3.1d and evaluated in Section 3.2.

d. Overtopping Potential

Both the Probable Maximum Flood and the half Probable Maximum Flood when routed through the reservoir, resulted in overtopping of the dam. The peak inflows for the PMF and one-half of the PMF are 887 cfs and 444 cfs, respectively. The peak outflow discharges for the PMF and one-half of the PMF are 616 and 50 cfs, respectively. The maximum capacity of the spillway just before overtopping the dam is 42 cfs. The PMF and one-half of the PMF overtopped the dam by 0.92 foot and 0.06 foot respectively. The total duration of flow over the dam is 4.33 hour and 1.58 hour for the PMF and the one-half of the PMF, respectively. Since the overtopping depth is only 0.06 feet during the occurrence of one-half of the PMF, the reservoir/spillway system of Stephens Lake Dam is considered capable of accommodating a flood equal to approximately 50 percent of the PMF just before overtopping the dam. The reservoir/spillway system of Stephens Lake Dam will accommodate the one-percent chance flood without overtopping. The surface soils in the embankment appear to be silty clay. The dam may be susceptible to erosion during overtopping.

The failure of the dam could cause extensive damage to the property downstream of the dam and possible loss of life. The estimated damage zone extends approximately four miles downstream of the dam. There are thirteen dwellings, a trailer court, several apartment houses and commercial buildings within the damage zone.

SECTION 6: STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

a. Visual Observations

There were no major signs of settlement or distress observed on the embankment or foundation during the visual inspection. The downstream slope of the embankment appears to be adequately protected from surface runoff erosion by a good grass cover. The erosion due to wave action on the upstream slope could affect the stability of the dam, if allowed to continue. There was no indication of past or present slope instability. In the absence of seepage and stability analyses, no quantitative evaluation of the structural stability can be made.

The stability of the spillway pipe appears to be questionable due to the misalignment of the joints, which appears to be due to the possible past piping of embankment material along the perimeter of the pipe, as described in Section 3.2. This condition, if allowed to worsen, will not only jeopardize the stability of the spillway further but will also jeopardize the stability of the dam.

b. Design and Construction Data

No design computations were uncovered during the report preparation phase. Seepage and stability analyses comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available. No embankment or foundation soil parameters were available for carrying out a conventional stability analysis on the embankment. No construction data or specifications relating to the degree of embankment compaction were available for use in a stability analysis.

c. Operating Records

No operating records were available relating to the dam or appurtenant structures. The water level on the day of the visual inspection was approximately 8 inches below the spillway crest. The normal operating level is considered to be at the spillway crest. According to Mr. Henley, the highest water level in the lake was approximately 3 inches above the spillway crest.

d. Post Construction Changes

According to Mr. Henley, two post construction changes have been made to the embankment since the original construction. The height of the dam was increased in 1939 to increase the reservoir capacity, and the existing spillway structure was installed in 1955 along the left side of the dam. It is unknown what effect these post construction changes had on the stability of the dam, if any.

e. Seismic Stability

The dam is located in Seismic Zone 1, as defined in "Recommended Guidelines for Safety Inspection of Dams" prepared by the Corps of Engineers, and will not require a seismic stability analysis. An earthquake of the magnitude which would be expected in Seismic Zone 1 will not cause distress to a well designed and constructed earth dam. Available literature indicates that no active faults exist near the vicinity of the damsite.

SECTION 7: ASSESSMENT/REMEDIAL MEASURES

7.1 Dam Assessment

The assessment of the general condition of the dam is based upon available data and visual inspection. Detailed investigations, testing and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

It should be realized that the reported condition of the dam is based upon observations of field conditions at the time of inspection along with data available to the inspection team.

It is also important to note that the condition of a dam depends upon numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be assurance that an unsafe condition could be detected.

a. Safety

The dam appears to be in fair physical condition due to the evidence of past piping of embankment materials along the spillway pipe. Also, the spillway capacity of Stephens Lake Dam is found to be "Inadequate". The spillway/reservoir system will accommodate approximately ⁴⁶ 50 percent of the PMF without overtopping the dam. The surface soils in the embankment appears to be silty clay. The dam embankment has a good grass cover. The dam is overtopped by 0.92 feet during the occurrence of the PMF. The dam may be susceptible to erosion due to overtopping of the dam during the PMF.

A quantitative evaluation of the safety of the embankment could not be made in view of the absence of seepage and stability analyses. The present embankment and appurtenant structures, however, reportedly have performed satisfactorily since their construction; there have been no failures. Reportedly, the dam has never been overtopped and no evidence indicating the contrary was observed. The safety of the dam can be improved if the deficiencies described in Section 3.2 and 6.1a are properly corrected as described in Section 7.2.

b. Adequacy of Information

The conclusions presented in this report are based upon field measurement, past performance and the present condition of the dam. Information on the design hydrology and hydraulic design of the dam was not available. Seepage and stability analyses comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available, which is considered a deficiency.

c. Urgency

The items recommended in paragraph 7.2a and the first item in paragraph 7.2b should be pursued on a high priority basis. The remaining remedial measures recommended in Paragraph 7.2 should be accomplished within a reasonable period of time.

d. Necessity for Phase II Inspection

Based upon results of the Phase I inspection, a Phase II inspection is not felt to be necessary.

7.2 Remedial Measures

a. Alternatives

One of the following mitigation measures should be undertaken under the guidance of an engineer experienced in the design and construction of earth dams to avoid severe consequences of dam failure from overtopping.

1. Increase the spillway capacity to pass the PMF without overtopping the dam.
2. Increase the height of the dam enough to pass the PMF without overtopping the dam; an investigation should also be done which includes studying the effects on the structural stability of the existing embankment. The overtopping depth during the occurrence of the PMF, stated in Section 5.1d, is not the required or recommended increase in the height of the dam.
3. A combination of 1 and 2 above.
4. Provide a highly reliable flood warning system (generally does not prevent damage but avoids loss of life).

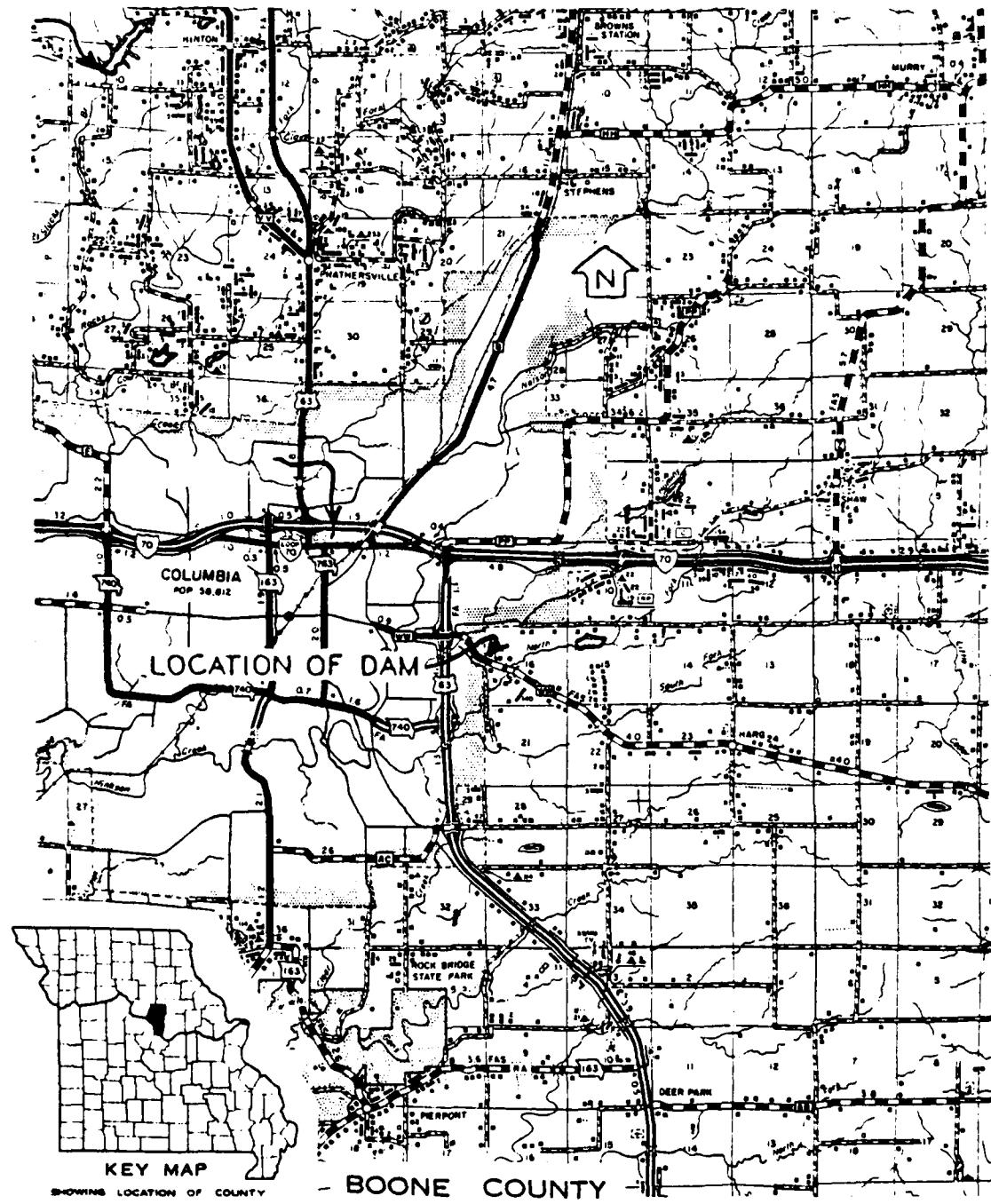
b. O & M Procedures

1. Further investigation should be undertaken to determine if indeed past piping of the embankment material has occurred along the spillway pipe. Measures should then be undertaken to control the condition and proper repairs made to correct the damages that have already occurred to the dam and the spillway. The investigation should be carried out under the direction of a qualified professional engineer.

2. The erosion due to wave action on the upstream slope should be properly repaired and adequately protected from further damage.
3. The small shrubs which are growing on the eroded area should be cleared from the embankment and prevented from growing back.
4. Determine the extent of damage done to the embankment by burrowing animals, if any, and make corrective repairs as required. All burrowing animals should be eliminated from the embankment and their burrows properly backfilled and compacted.
5. Seepage and stability analyses should be performed by a professional engineer experienced in the design and construction of earth dams.
6. The owner should initiate the following programs:
 - (a) Periodic inspection of the dam by a professional engineer experienced in the design and construction of earth dams.
 - (b) Set up a maintenance schedule and log all visits to the dam for operation, repairs and maintenance.

PLATES

PLATE 1



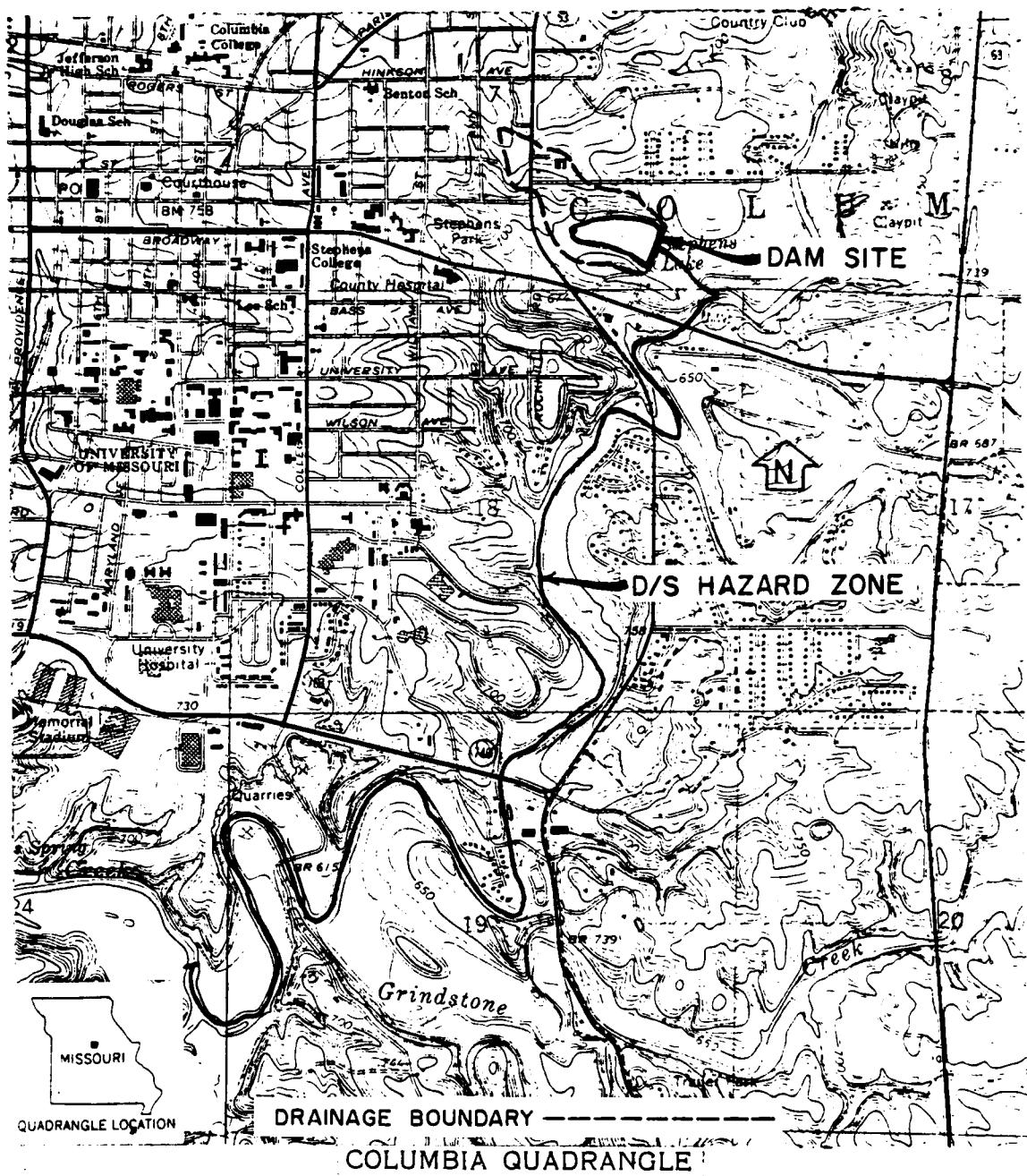
SCALE
1 0 1 2 3 4 MILES

POLYCONIC PROJECTION

LOCATION MAP - STEPHENS LAKE DAM

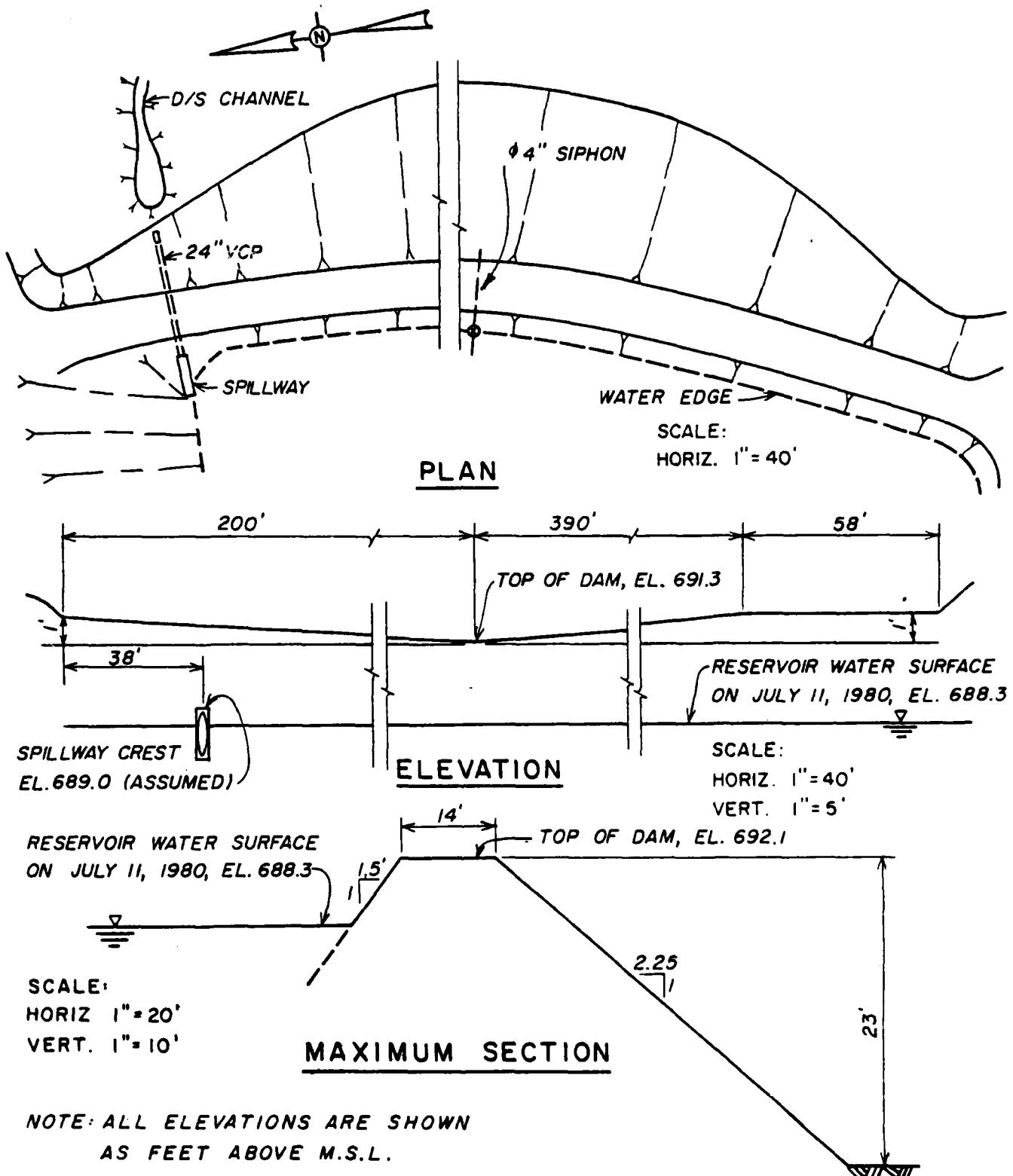
MO.11172

PLATE IA



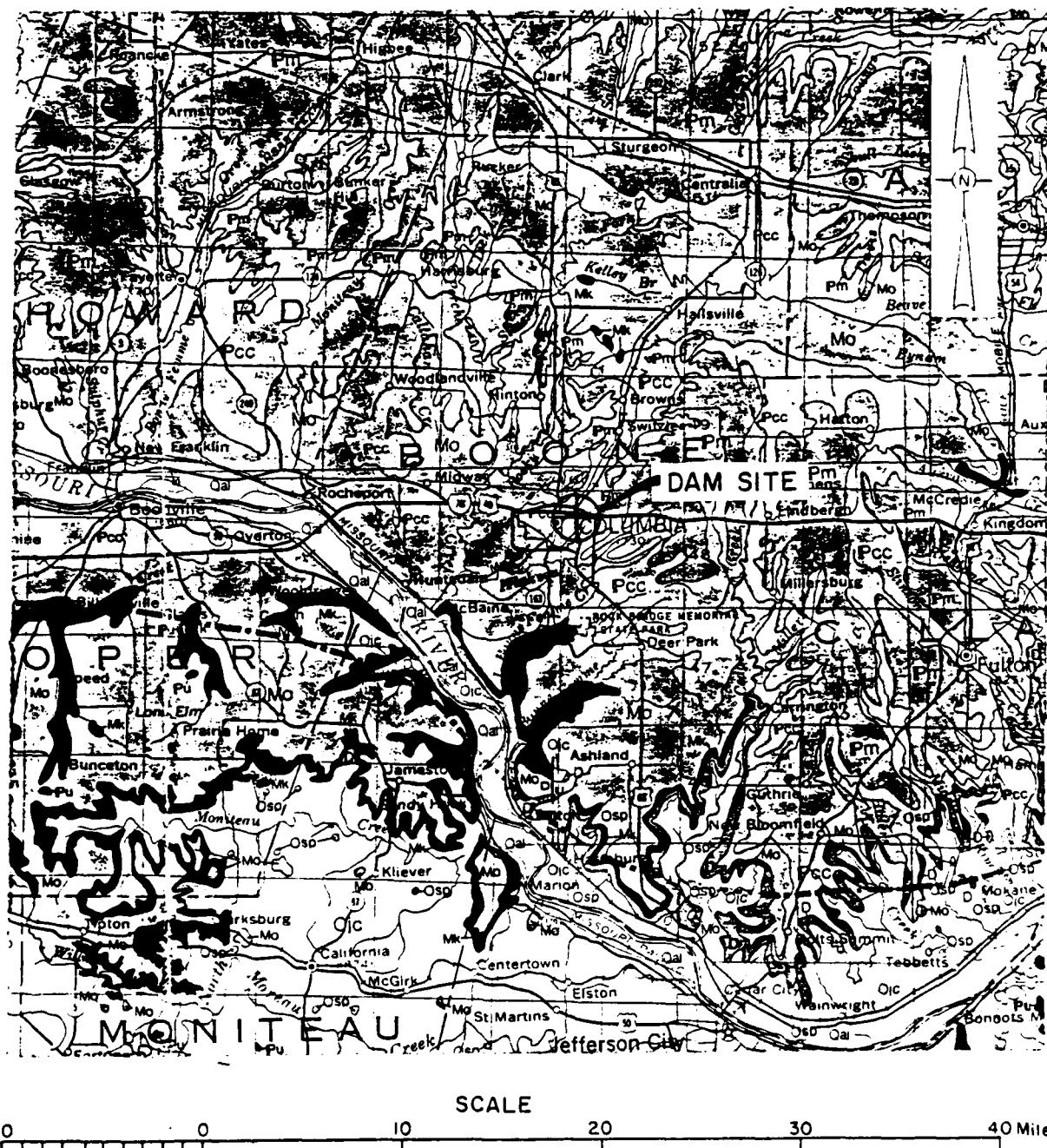
STEPHENS LAKE DAM (MO. #172)
DRAINAGE BASIN AND
DOWNSTREAM HAZARD ZONE

PLATE 2



STEPHENS LAKE DAM (MO. 11172)
PLAN, ELEVATION &
MAXIMUM SECTION OF EMBANKMENT

PLATE 3



SCALE

10 0 10 20 30 40 Miles

LOCATION OF DAM

NOTE: LEGEND OF THIS DAM IS ON PLATE 4

REFERENCE:

GEOLOGIC MAP OF MISSOURI
DEPARTMENT OF NATURAL RESOURCES
MISSOURI GEOLOGICAL SURVEY
KENNETH H. ANDERSON, 1979

REGIONAL GEOLOGICAL MAP
OF
STEPHENS LAKE DAM

STEPHENS LAKE DAM

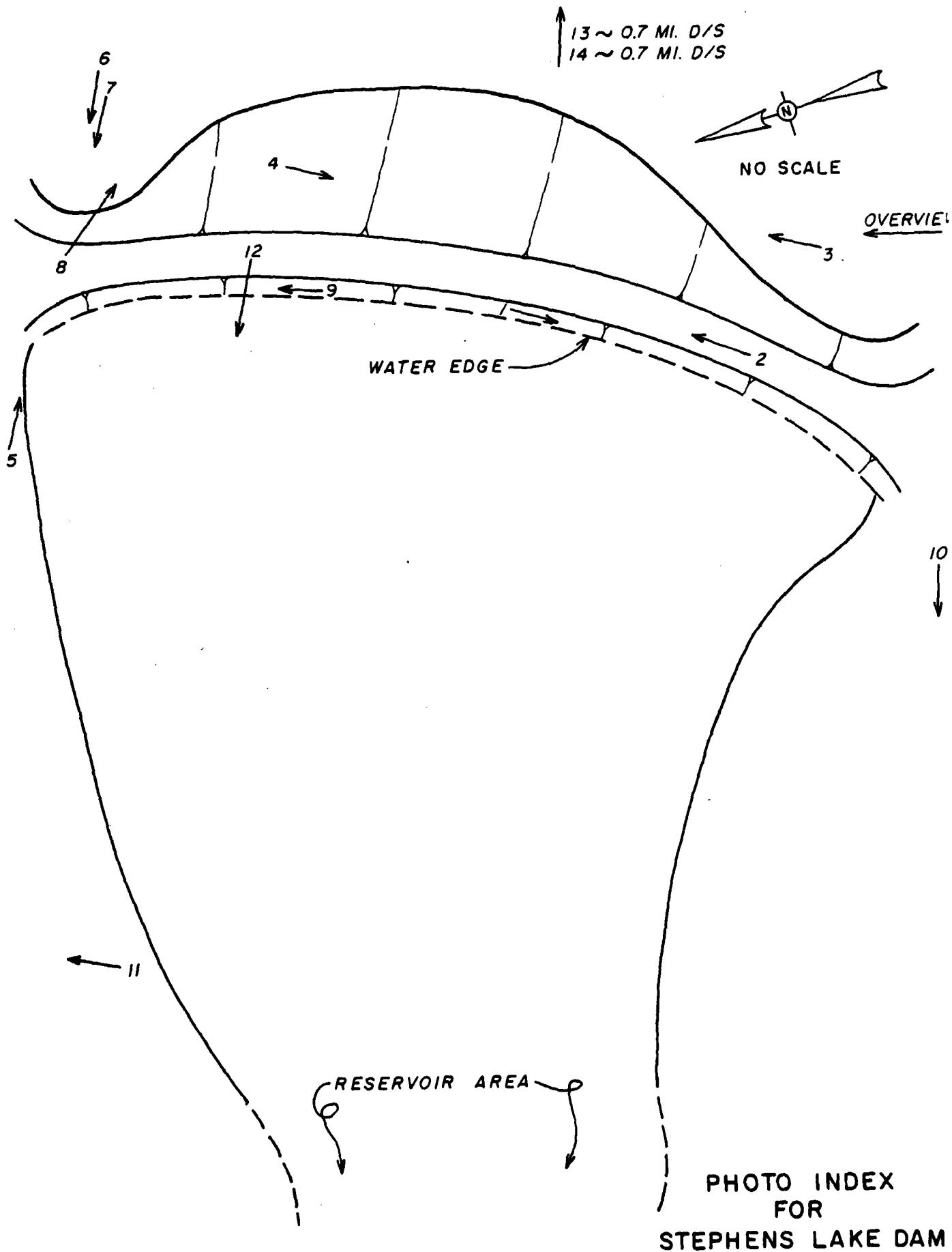
PLATE 4

LEGEND

<u>PERIOD</u>	<u>SYMBOL</u>	<u>DESCRIPTION</u>
QUATERNARY	Qal	ALLUVIUM: SAND, SILT, GRAVEL
PENNSYLVANIAN	Pu	PENNSYLVANIAN UNDIFFERENTIATED
	Pm	MARMATON GROUP: CYCLIC DEPOSITS OF SHALE, LIMESTONE AND SANDSTONE
	Pcc	CHEROKEE GROUP: CYCLIC DEPOSITS OF SHALE, LIMESTONE AND SANDSTONE
MISSISSIPPIAN	Mo	KEOKUK - BURLINGTON FORMATION: CHERTY GRAYISH BROWN SANDY LIMESTONE
	Mk	CHOUTEAU GROUP: NORTHVIEW AND BACHELOR FORMATION (LIMESTONE AND SHALE)
DEVONIAN	D	SULPHUR SPRING GROUP: GLEN PARK LIMESTONE AND GRASSY CREEK SHALE
ORDOVICIAN	Osp	ST PETER SANDSTONE
	Ojc.	SMITHVILLE FORMATION POWELL DOLOMITE

APPENDIX A

PHOTOGRAPHS



Stephens Lake Dam

Photographs

Photo 1 - View of the upstream slope showing vegetative cover and riprap. Note the batter boards holding the riprap in place.

Photo 2 - View of the top of dam.

Photo 3 - View of the downstream slope.

Photo 4 - View of a rodent hole on the downstream slope.

Photo 5 - View of the concrete side channel structure and the inlet to the clay pipe. Note the dumped concrete to the right of the side channel.

Photo 6 - View of the outlet of the clay pipe showing the concrete apron, the 5-foot diameter steel drum and the riprap in the discharge channel. Note the depression on the slope behind and to the left (in photo) of the outlet.

Photo 7 - Close-up view of the depression in Photo 6 showing erosion of the embankment material along the spillway outlet pipe.

Photo 8 - View of the downstream channel from the left abutment.

Photo 9 - View of the 4-inch siphon on the upstream slope.

Photo 10 - View of the vertical centrifugal turbine pump located on the right side of the reservoir rim.

Photo 11 - View of the limestone outcrop on the northeast side of the reservoir.

Photo 12 - View of the reservoir and rim.

Photo 13 - View of a dwelling downstream of the dam that appears to be in the downstream hazard zone.

Photo 14 - View of a dwelling downstream of the dam that appears to be in the downstream hazard zone.

Stephens Lake Dam



Photo 1



Photo 2

Stephens Lake Dam



Photo 3

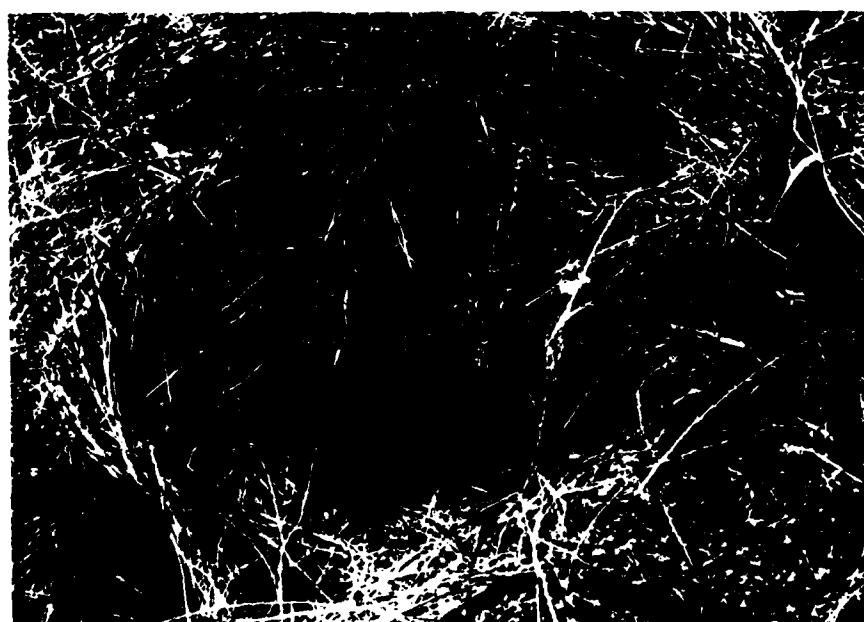


Photo 4

Stephens Lake Dam



Photo 5



Photo 6

Stephens Lake Dam



Photo 7



Photo 8

Stephens Lake Dam



Photo 9



Photo 10

Stephens Lake Dam



Photo 11



Photo 12

Stephens Lake Dam



Photo 13



Photo 14

APPENDIX B
HYDROLOGIC AND HYDRAULIC COMPUTATIONS

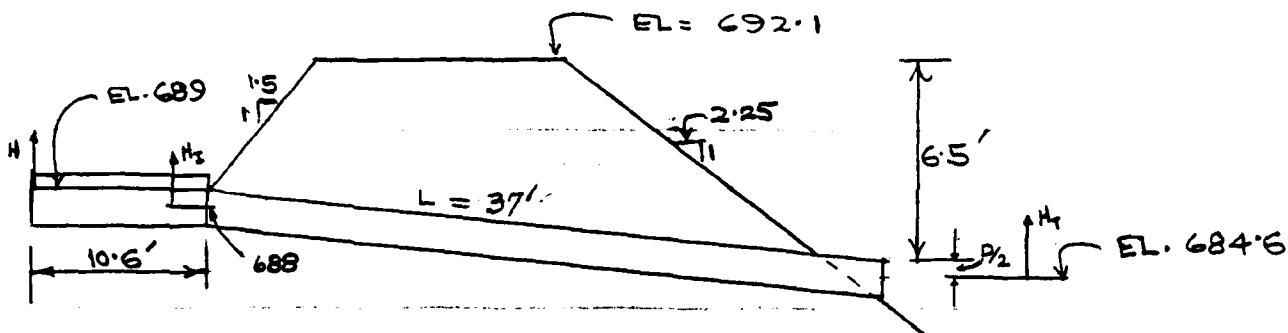
STEPHENS LAKE DAM

HYDROLOGIC AND HYDRAULIC DATA, ASSUMPTIONS AND METHODOLOGY

1. SCS Unit Hydrograph and HEC-1DB are used to develop the inflow hydrographs, and the hydrologic inputs are as follows:
 - (a) Twenty-four hour probable maximum precipitation from Hydro-meteorological Report No. 33, and 100-year 24-hour rainfall of Jefferson City, Missouri.
 - (b) Drainage area = 38 acres.
 - (c) Lag time = 0.07 hour.
 - (d) Hydrologic Soil Group:
Soil Group "C"
 - (e) Runoff curve number:
CN = 80 for AMC II and CN = 91 for AMC III.
2. Spillway release rates are based on weir, orifice, and pressure flow depending on the stage of the reservoir. Flow rates over the dam are based on broad crested weir equation $Q = CLH^{3/2}$ and critical depth assumption.
3. Floods are routed through Stephens Lake to determine the capability of its spillway.

ECI-4 PRC ENGINEERING CONSULTANTS, INC.

DAM SAFETY INSPECTION / MISSOURI - 1980 SHEET NO. 1 OF 4
 STEPHENS LAKE DAM (MO. 11172) JOB NO. 1263
 SPILLWAY DISCHARGE COMPUTATIONS BY MAS DATE 8/27/80



Weir Flow:

$$Q = CLH^{1.5}, C = 3.0 \\ L = 10.6' \\ H = \text{W.S. EL} - 689$$

$$Q = 3.0(10.6)(H)^{1.5} \\ Q = 31.80(H)^{1.5}$$

Orifice Flow:

$$Q = CA\sqrt{2gH_x}, C = 0.84 \\ A = \pi \\ H_x = \text{W.S. EL} - 688$$

$$Q = 0.84(\pi)\sqrt{2gH_x} \\ Q = 21.18\sqrt{H_x}$$

Pressure Flow:

$$Q = A\sqrt{\frac{2g}{\Sigma K} H_t}, A = \pi$$

$\Sigma K = (K_{entrance} + K_{friction} + K_{exit})$, where

$$K_{entrance} = 0.5$$

$$K_{friction} = \frac{29.16 \frac{m^2 L}{(R_N)^{4/3}}}{(0.018)^2 (37')} = \frac{29.16 (0.018)^2 (37')}{(0.5)^{4/3}}, \quad 0.88$$

$$K_{exit} = 1.0$$

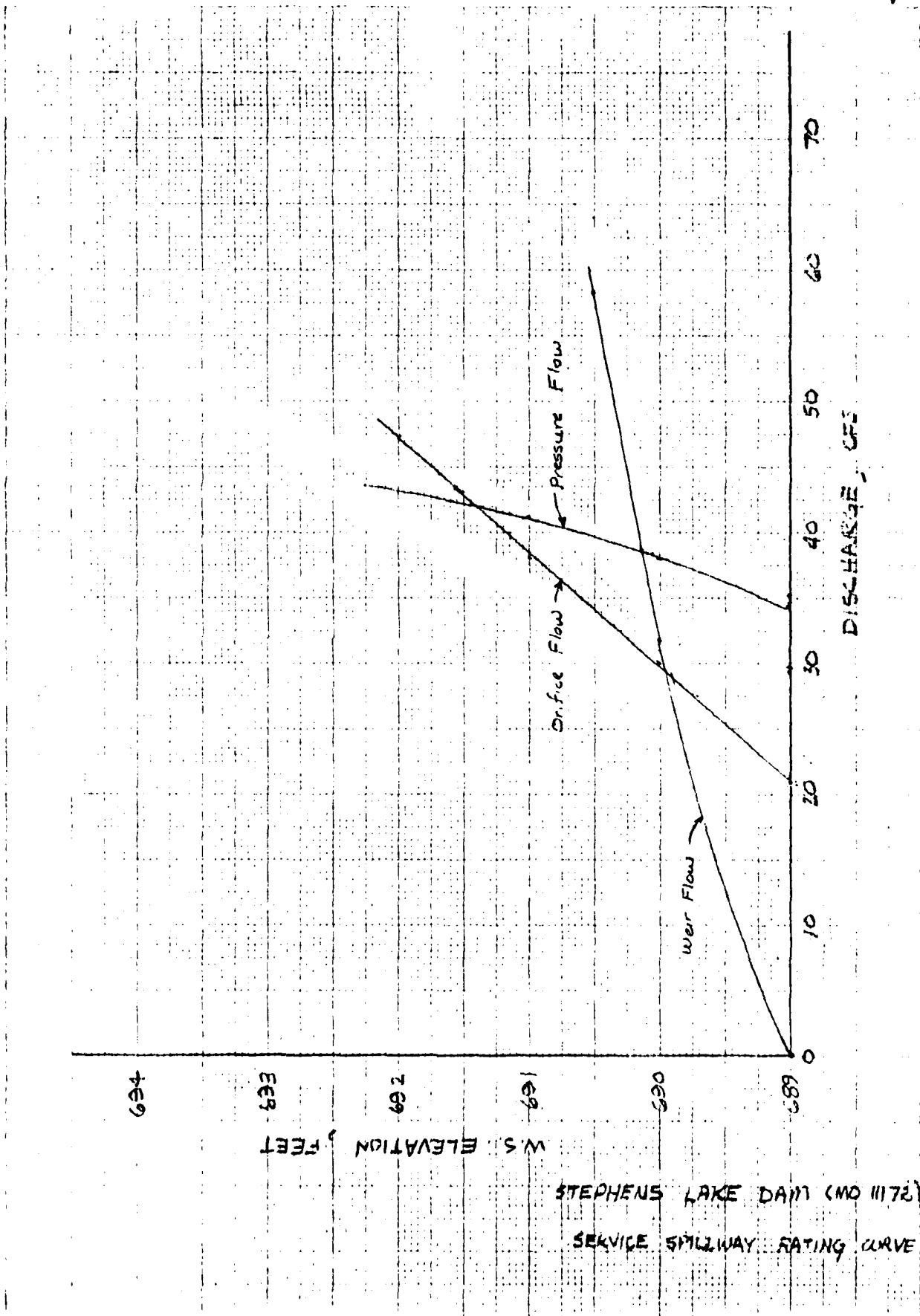
$$\Sigma K = 2.38$$

$$Q = \pi \sqrt{\frac{2g}{2.38} H_t} = 16.34 \sqrt{H_t}, \quad H_t = \text{W.S. EL} - 684.6$$

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46 1621

244



ECI-4 PRC ENGINEERING CONSULTANTS, INC.

DAM SAFETY INSPECTION - MISSOURI

STEPHENS LAKE DAM (No 11172)

OVERTOP RATING CURVE

SHEET NO. 3 OF 4

JOB NO. 1263

BY D.C. DATE 7/27/80
ALB



$H_1 = \frac{Y_1}{(1 + \frac{\Delta Y_1}{2})}$	$T_1 = A_1 Q \cdot \sqrt{\frac{A_1^3 g}{T_1}}$	$A_2 = C_2 \cdot \sqrt{\frac{A_2^3 g}{T_2}}$	C_3	$L_3 = H_3 \cdot C_3 \cdot C_{41} \cdot C_{42} \cdot C_{43} \cdot C_{44} \cdot C_{45}$	$WSEL$				
.3	.24	93.6	11.23	.24	48	5.74	1.32	33	691.6
.7	.56	218.4	6.115	133.67	.56	112	31.36	4.16	692.0
1.0	.80	312.0	1.248	47.89	.80	160	64	29.69	692.3
1.3	1.03	310	208.6	81.97	1.03	200	104.67	42.03	29.58
1.7	1.30	390	312.0	193.5	1.30	200	160	82.07	3.03
2.2	1.63	390	442.0	200.1	1.63	200	226.67	13.93	3.04
2.7	1.97	390	572.0	330.9	1.97	200	293.32	4.58	3.04
3.0	2.17	390	650.0	411.7	217	200	333.32	11.1	3.05
3.4	2.43	390	754.0	519.12	243	200	386.67	350.8	3.06
3.7	2.63	390	832.0	615.7	263	200	426.73	336.3	3.07

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ECI-4 PRC ENGINEERING CONSULTANTS, INC.

DAM SAFETY INSPECTION / MISSOURI - 1980

SHEET NO. 4 OF 4

STEPHEN'S LAKE DAM (MO 11172)

JOB NO. 1263

SPILLWAY AND OVERTOP RATING CURVE

BY JFK DATE 9/2/80

W.S. ELEV.	Q SERVICE SPWY.	Φ OVERTOP	Φ COMBINED
689	0		0
690	31.8*		31.8
691	36.7**		36.7
691.3	42.3***	0	42.3
691.6	43.2	33	76
692	44.5	278	323
692.3	45.3	678	723
692.6	46.2	1332	1378
693	47.4	2498	2546
693.5	48.8	4271	4320
694	50.1	6338	6388
694.3	50.9	7703	7754
694.7	51.9	9660	9712
695	52.7	11221	11274

* Weir flow controls

** Orifice flow controls

*** Pressure flow controls at EL = 691.3 and above

ECI-4 PRC ENGINEERING CONSULTANTS , INC.

DAM SAFETY INSPECTION - MISSOURI

SHEET NO. _____ OF

DAM NAME: STEPHENS LAKE DAM / ID NO.: 11172

JOB NO.

RESERVOIR ELEVATION - AREA DATA

BY D.C.

K-3

DATE 7/8/80

ELEV. (M.S.L.) (Ft.)	RESERVOIR SURFACE AREA (Acres)	REMARKS
670.	0	Estimated ELEVATION
689	10	Spillway weir crest (Assumed)
690	11	Measured on USGS Map
691.3	12	Top of dam (minimum)
700	16.5	Measured on USGS Map
710	19.0	Measured on USGS Map

ECI-4 PRC ENGINEERING CONSULTANTS, INC.

DAM SAFETY INSPECTION / MISSOURI

SHEET NO. 1 OF 1

DAM NAME: STEPHENS LAKE DAM (MO 11172)

JOB NO. 1263

UNIT HYDROGRAPH PARAMETERS

BY D.C. DATE 7/24/80

KLE

1) DRAINAGE AREA, $A = .059 \text{ sq. mi} = (38.0 \text{ acres})$ 2) LENGTH OF STREAM, $L = (.75 \text{ "} \times 2000 \text{ '} = 1500 \text{ '}) = .284 \text{ mi.}$

3) ELEVATION AT DRAINAGE DIVIDE ALONG THE LONGEST STREAM,

$$H_1 = 777'$$

4) ELEVATION OF RESERVOIR AT SPILLWAY CREST, $H_2 = 689'$ 5) ELEVATION OF CHANNEL BED AT $0.85L$, $E_{85} = 768'$ 6) ELEVATION OF CHANNEL BED AT $0.10L$, $E_{10} = 698'$ 7) AVERAGE SLOPE OF THE CHANNEL, $S_{AVG} = (E_{85} - E_{10}) / 0.75L = .062$

8) TIME OF CONCENTRATION:

A) BY KIRPICH'S EQUATION,

$$t_c = [(11.9 \times L^3) / (H_1 - H_2)]^{0.385} = \left[\frac{11.9 (.284)^3}{777-689} \right]^{0.385} = .11$$

B) BY VELOCITY ESTIMATE,

$$\text{SLOPE} = 6.2\% \Rightarrow \text{AVG. VELOCITY} = 5 \text{ fps}$$

$$t_c = L/v = 1500/5(3600) = .083$$

$$\text{USE } t_c = .11$$

9) LAG TIME, $t_l = 0.6 t_c = .066 \approx .07$ 10) UNIT DURATION, $D \leq t_c / 3 = .022 < 0.083 \text{ hr.}$

$$\text{USE } D = .083$$

11) TIME TO PEAK, $T_p = D/2 + t_c = .107$

12) PEAK DISCHARGE,

$$q_p = (484 \times A) / T_p = 266 \text{ cfs}$$

ECI-4 PRC ENGINEERING CONSULTANTS, INC.

DAM SAFETY INSPECTION

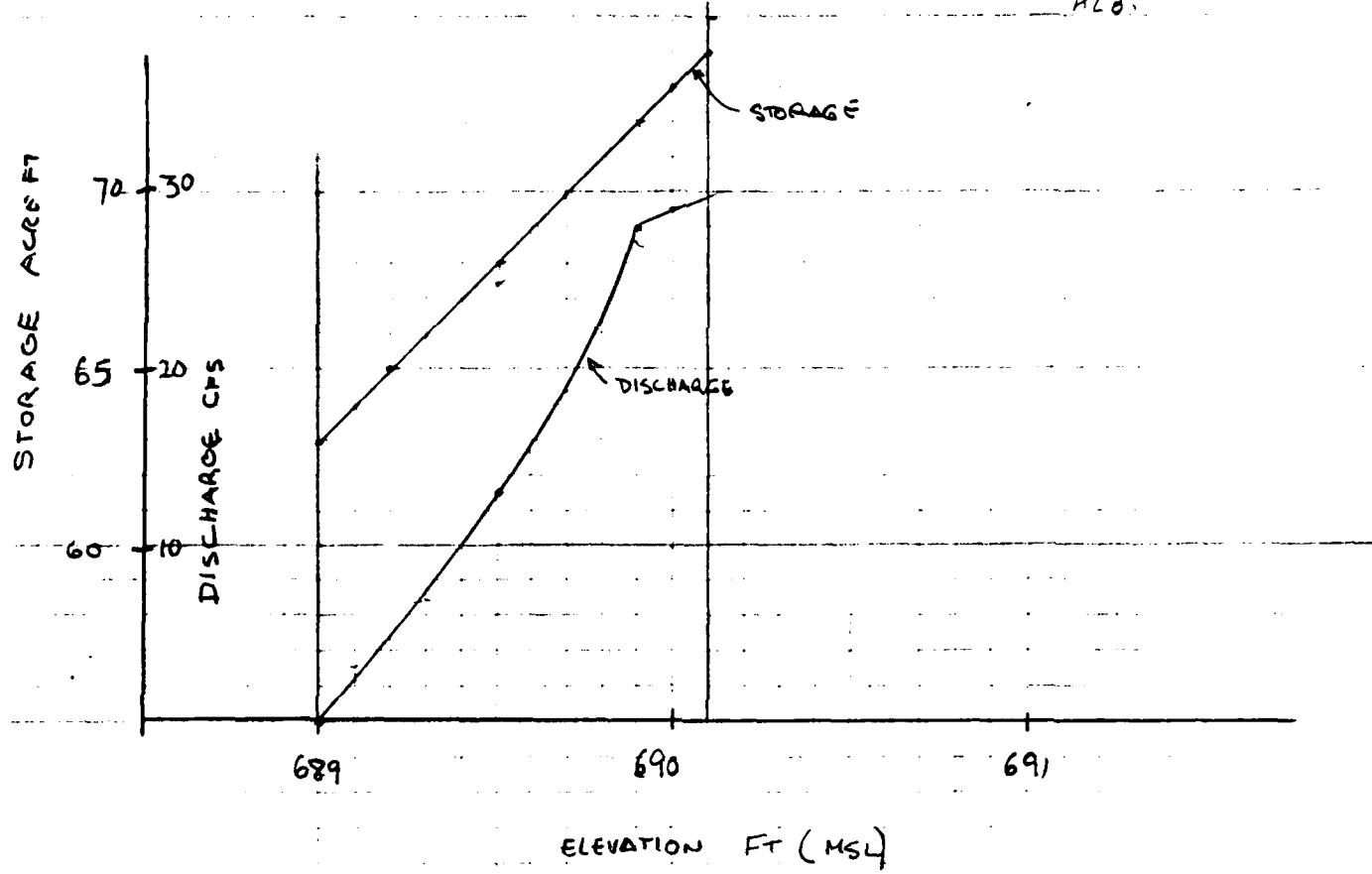
SHEET NO. _____ OF _____

STEPHENS LAKE DAM (MO 11172)

JOB NO. 1263

STARTING WSL FOR PMF ROUTING

BY DC DATE 7/30/80
HLD.



From 690.1 - 689.5

$$\Delta S = 74 - 68 = 6 \text{ acre ft}$$

$$Q_m = 22 \text{ cfs}$$

$$\Delta t = 6 \text{ acre ft} \times \frac{43560 \text{ ft}^2}{\text{acre}} \times \frac{1 \text{ s}}{22 \text{ ft}^3} \times \frac{1 \text{ day}}{86400 \text{ s}} = .1375$$

From 689.5 - 689

$$\Delta S = 68 - 63 = 5 \text{ acre ft}$$

$$Q_m = 7 \text{ cfs}$$

$$\Delta t = .36 \text{ days}$$

$$\text{total time} = .1375 + .36 = .5 \text{ days} < 3 \text{ days}$$

\therefore Start PMF routing at spillway crest

FLOOD HYDROGRAPH PACKAGE, INTRACOASTAL
WATERWAY, MILE 100, JULY 1971
LAST MODIFICATION, 126 FEB 7

DAM SAFETY INSPECTION MISSOURI
CYPHERSTEAK-DOH FRO (LIVY)

MONDAY 6
TUESDAY 5
WEDNESDAY 4
THURSDAY 3
FRIDAY 2
SATURDAY 1

THE PRACTICAL WORKER

SUB-AREA HUNGER COMPUTATION

INPUT PRECIPITATION, INFLUX RATIO, AND UNIT HYDROGRAPH PARAME

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CURVE NO G 391.00 WINFSIG = -1.00 EFFECT CN = -1.00
UNIT HYDROGRAPH DATA
TCB: 0.00 LAGE: .07

TIME INCREMENT TOO LARGE--INFO IS BY LAG(2)
LINE 1 HYDROGRAPH & END OF PERIOD ORDINATES. TIME
LINE 1

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B-11

B-14

DETERMINED STREAMFLOW SUMMARY FOR UNITLESS PLAN-STATIC CONDUCTIONS
FLOWS IN CUBIC FEET PER SECOND (CFS) METER PER SECOND,
AREA IN SQUARE MILES (SQUARE KILOMETERS)

RATINGS APPLICABLE FLOWS
PER FLOW STATION AREA PLAN METER OF WATER

1.00 .50

MICROGRAPH AT	011172	.0036	1	497	.004
		.0150	1	25.1234	12.5604
ROUTE TO	011172	.05	1	615	.50
		.10	1	17.4514	1.0004

DISCUSSIONS AND CONCLUSIONS

MAXIMUM RESERVOIR LEVEL		MAXIMUM STORAGE AC-FT		MAXIMUM OUTLF.		MAXIMUM OVER TOP AC-FT		MAXIMUM TIME	
DEPTH OF WATER	DEPTHS OF WATER	DEPTH OF WATER	DEPTHS OF WATER	DEPTH OF WATER	DEPTHS OF WATER	DEPTH OF WATER	DEPTHS OF WATER	TIME	TIME
699.00	699.00	699.00	699.00	699.00	699.00	699.00	699.00	15.0	15.0
698.00	698.00	698.00	698.00	698.00	698.00	698.00	698.00	14.9	14.9
697.00	697.00	697.00	697.00	697.00	697.00	697.00	697.00	14.8	14.8
696.00	696.00	696.00	696.00	696.00	696.00	696.00	696.00	14.7	14.7
695.00	695.00	695.00	695.00	695.00	695.00	695.00	695.00	14.6	14.6
694.00	694.00	694.00	694.00	694.00	694.00	694.00	694.00	14.5	14.5
693.00	693.00	693.00	693.00	693.00	693.00	693.00	693.00	14.4	14.4
692.00	692.00	692.00	692.00	692.00	692.00	692.00	692.00	14.3	14.3
691.00	691.00	691.00	691.00	691.00	691.00	691.00	691.00	14.2	14.2

ITEM	TIME OF TEST	TIME OF FAILURE	MAX OUTFLW HOURS	FAILURE HOURS
S	15.15	2.00	0.00	0.00

B-18

HYDROGRAPH FOR 2 AND 4 STORAGES (END OF PERIOD) SUMMARY FROM MULTIPLE ECONOMIC COMPUTATIONS
FLOOD IN FEET PER SECOND (CUBIC FEET PER SECOND)
AREA IN SQUARE MILES (SQUARE MILLEFEET)

EFFICIENCY - AREA = PLAN RATIO = RATIO OF AREA .40 .45 .50

HYDROGRAPH NO.	STATION	TIME	RATIO	AREA	FLOOD	EFFICIENCY
1	011172	0	.06	1	355.	.399.
2	011172	6	.15	1	19.0530	11.3504

SUMMARY OF DAM BREAK ANALYSIS

ELEVATION	INITIAL VALUE	TOP OF DAM
689.00	689.00	691.30
6.3	6.3	6.9
OVERFLOW	OVERFLOW	OVERFLOW

RATIO	MATERIAL	MAXIMUM	MAXIMUM	DURATION
OF	INFILTRATION	DEPTH	OUTFLOW	TIME OF
P.H.	W.S.ELEV.	OVER DAM	CFS	OVER TOP
1.00	694.92	0.00	0.00	0.00
0.98	694.92	0.02	0.02	0.02
0.96	694.92	0.06	0.06	0.06
0.95	694.92	0.08	0.08	0.08

RATIO	MATERIAL	MAXIMUM	MAXIMUM	DURATION
OF	INFILTRATION	DEPTH	OUTFLOW	TIME OF
P.H.	W.S.ELEV.	OVER DAM	CFS	OVER TOP
1.00	694.92	0.00	0.00	0.00
0.98	694.92	0.02	0.02	0.02
0.96	694.92	0.06	0.06	0.06
0.95	694.92	0.08	0.08	0.08

RATIO	MATERIAL	MAXIMUM	MAXIMUM	DURATION
OF	INFILTRATION	DEPTH	OUTFLOW	TIME OF
P.H.	W.S.ELEV.	OVER DAM	CFS	OVER TOP
1.00	694.92	0.00	0.00	0.00
0.98	694.92	0.02	0.02	0.02
0.96	694.92	0.06	0.06	0.06
0.95	694.92	0.08	0.08	0.08